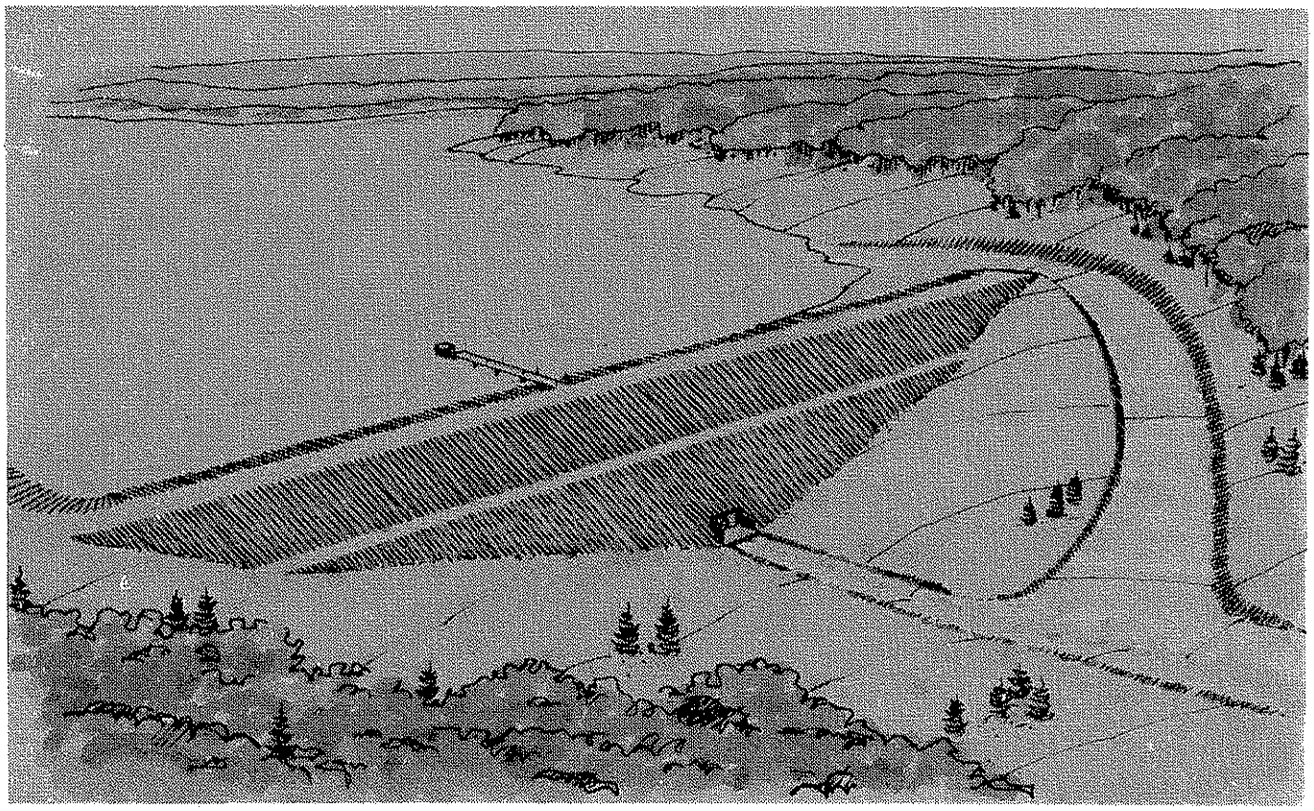

Dam Safety Guidelines

Technical Note 1:
Dam Break Innundation Analysis and Downstream Hazard Classification

WASHINGTON STATE DEPARTMENT OF ECOLOGY

July 1992
92-55E

printed on recycled paper

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Dam Safety Guidelines

Technical Note 1: *Dam Break Innundation Analysis and Downstream Hazard Classification*

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July 1992
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DAM BREAK INUNDATION ANALYSIS

1. INTRODUCTION

The outflow flood hydrograph from a dam failure is dependent upon many factors. The primary factors are the physical characteristics of the dam, the volume of the reservoir and the mode of failure. The parameters which control the magnitude of the peak discharge and the shape of the outflow hydrograph include: the breach dimensions; the manner and length of time for the breach to develop; the depth and volume of water stored in the reservoir; and the inflow to the reservoir at the time of failure. The shape and size of the breach and the elapsed time of development of the breach are in turn dependent upon the geometry of the dam, construction materials, and the causal agent for failure.

The field of dam breach inundation analysis is relatively recent and most advances have occurred since about 1977. Because of the many recent advances, there is value in reviewing procedures and concepts which were initially proposed and how the methodologies have evolved with time. Before proceeding with a discussion of the numerical methods currently available for conducting dam break inundation analyses, it is appropriate to present experiences gained from observed dam failures.

1.1 <u>CAUSES OF DAM FAILURE</u>

Information on the causal agents for dam failures has been collected since the 1850s. Technology has obviously changed drastically since that time and improved design standards and construction practices continue to reduce the number of failures. Nonetheless, the relative proportion of dam failures attributable to a specific cause have remained relatively constant over the years[1,12].

A study conducted by Middlebrooks[1] into the causes of 220 earth dam failures during the period 1850-1950 summarizes observed causal agents and their frequency of occurrence (Table 1). It is interesting to note that 50 percent of the failures catalogued by Middlebrooks occurred within the first five years and that 19 percent failed upon first filling (Table 2).

A review of Table 1 information indicates that one of two reservoir conditions commonly exist at the time of failure. For flood induced failures, the reservoir level would exceed the dam crest elevation. For other failure modes, such as induced by seepage, internal erosion, slope failure of the embankment under static or seismic loadings, the reservoir level is commonly at, or near, normal pool elevation.

For this reason, at least two reservoir conditions, normal pool and dam overtopping need to be examined as part of any dam break inundation analysis.

TABLE 1. CAUSES OF EARTH DAM FAILURES 1850-1950

CAUSE	SOURCE MECHANISM	% OF TOTAL
OVERTOPPING	FLOOD	30%
PIPING/INTERNAL EROSION OF EMBANKMENT OR FOUNDATION	SEEPAGE, PIPING AND INTERNAL EROSION	25%
CONDUIT LEAKAGE		13%
DAMAGE/FAILURE OF UPSTREAM MEMBRANE/SLOPE PAVING		5%
EMBANKMENT INSTABILITY-SLIDES	VARIES	15%
MISCELLANEOUS	VARIES	12%

TABLE 2. DAM FAILURES - AGE OF DAM AT TIME OF FAILURE

NUMBER OF YEARS AFTER COMPLETION	CAUSE OF FAILURE (%)				TOTAL %
	OVERTOPPING	CONDUIT LEAKAGE	SEEPAGE	SLIDES	
0-1	9	23	16	29	19%
1-5	17	50	34	24	31%
5-10	9	9	13	12	11%
10-20	30	9	13	12	16%
20-50	32	9	24	23	22%
50-100	3	0	0	0	1%

2. ESTIMATION OF DAM BREACH CHARACTERISTICS

2.1 ESTIMATION OF DAM BREACH DIMENSIONS

Prior to the 1980s, little analytical work had been done on numerically describing the mechanics of failure. During the early 1980's, estimation of dam breach dimensions were based solely on values from observed failures. Guidelines from the U.S. Army Corps of Engineers[2] (COE) and from Fread[3] with the National Weather Service (NWS) include recommendations for breach parameters as shown in Table 3.

TABLE 3. RECOMMENDED VALUES FOR BREACH PARAMETERS - CIRCA 1980

DAM TYPE	BREACH WIDTH	SIDE SLOPE OF BREACH	FAILURE TIME
EARTHFILL DAM	0.5 TO 3.0 DAM HEIGHTS	VERTICAL TO 1:1	0.5 TO 4.0 HRS (COE) 0.1 TO 2.0 HRS (NWS)
CONCRETE GRAVITY DAM	INTEGER MULTIPLE OF MONOLITH WIDTHS	VERTICAL	0.1 TO 0.5 HRS
CONCRETE ARCH DAM	ENTIRE VALLEY WIDTH	VALLEY WALL	0 TO 0.1 HRS

At that time, it was recognized that for earthfill dams, large breach dimensions were associated with poorly constructed dams, dams constructed of easily erodible materials and dams with large volumes of storage. Rapid failures were associated with easily eroded materials, concrete structures having the potential for brittle failures and causal agents which can trigger rapid failures. In the mid 1980s, MacDonald and Langridge-Monopolis[4], and Froelich[5] were successful in relating breaching characteristics of earthfill dams to physically measurable features of the dam and reservoir. Their work provided some predictive capability in estimating breach parameters. Specifically, the volume of material eroded in the breach was found to be related to the Breach Formation Factor (BFF):

$$BFF = V_w (H) \tag{1}$$

where:

V_w = Volume of water stored in the reservoir (acre-ft) at the water surface elevation under consideration

H = Height of water (ft) over the base elevation of the breach

Interpretation of the MacDonald and Langridge-Monopolis[4] data suggests that estimates of the volume of material eroded from earthen dams comprised of Cohesionless Embankment Materials may be taken to be:

$$V_m = 3.75\ (BFF)^{.77} \tag{2a}$$

and, for Erosion Resistant Embankment Materials;

$$V_m = 2.50\ (BFF)^{.77} \tag{2b}$$

where:

V_m = Volume of material in breach (yds^3) which is eroded.

These equations are graphically displayed in Figure 1. Gray tone areas have been added to reflect the scatter in observed values and the uncertainties involved in parameter estimation[4].

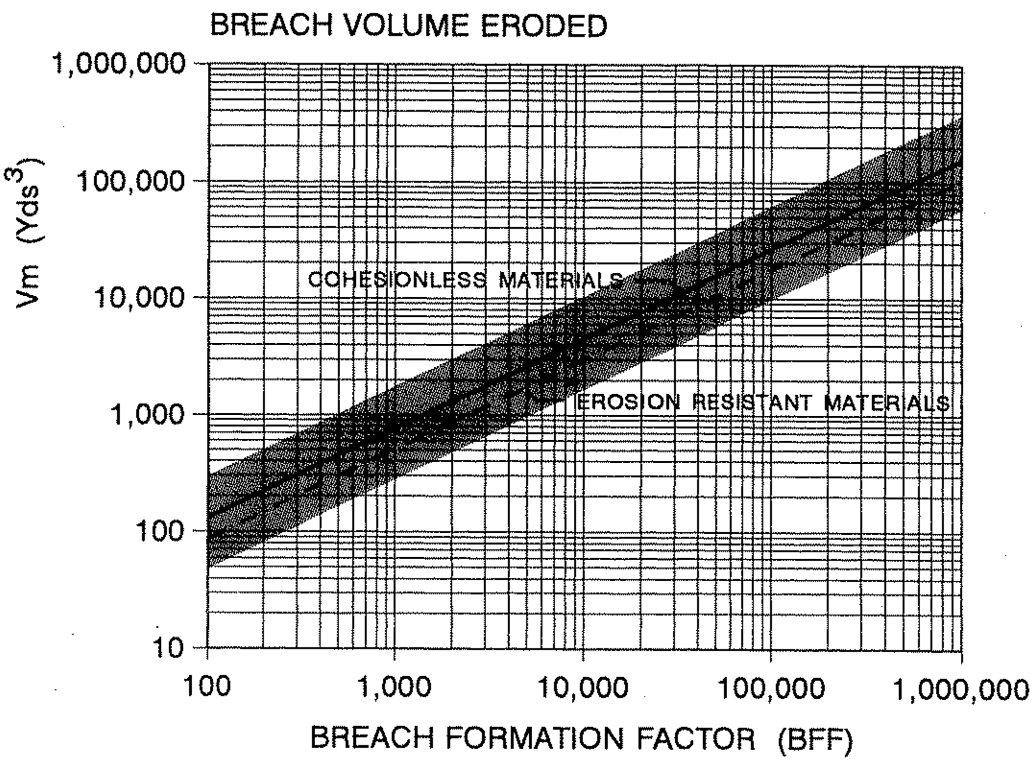

FIGURE 1 - ESTIMATED ERODED VOLUME OF BREACH

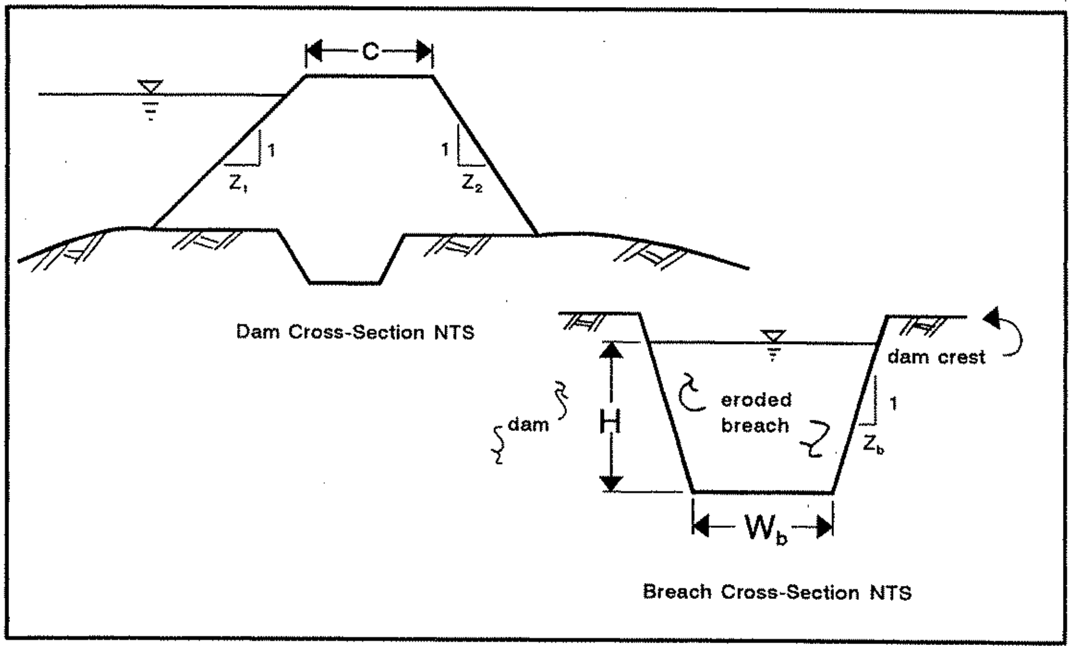

FIGURE 2 - DAM AND BREACH GEOMETRY

Experience has shown that breaches in earthen dams are generally trapezoidal in shape. The base elevation of the breach usually approximates the streambed elevation unless some site specific condition restricts erosion to some other elevation. Using the geometry of the dam and breach shown in Figure 2, the base width of the breach can be computed as a function of the eroded volume of material as:

For a rectangular breach, $Z_b = 0$

$$W_b = \frac{27\ V_m}{H(C + HZ_3/2)} \tag{3}$$

where:

W_b = Width of breach (ft) at base elevation of breach
C = Crest width of dam (ft)
$Z_3 = Z_1 + Z_2$ and;

Z_1 = Slope (Z_1:1) of upstream face of dam
Z_2 = Slope (Z_2:1) of downstream face of dam

For a trapezoidal breach with sideslopes of (Z_b:1)

$$W_b = \frac{27\ V_m - H^2\ (CZ_b + HZ_bZ_3/3)}{H(C + HZ_3/2)} \tag{4}$$

The elapsed time (τ) in hours, for breach development has been related to the volume of eroded material (V_m) by MacDonald and Langridge-Monopolis[4] as shown in Figure 3. Interpretation of their data suggests that the time for breach development can be estimated by:

$$\tau = .028\ V_m^{.36} \tag{5a}$$

for earthen dams of predominately cohesionless materials; and

$$\tau = .042\ V_m^{.36} \tag{5b}$$

for earthen dams of predominately erosion resistant materials.

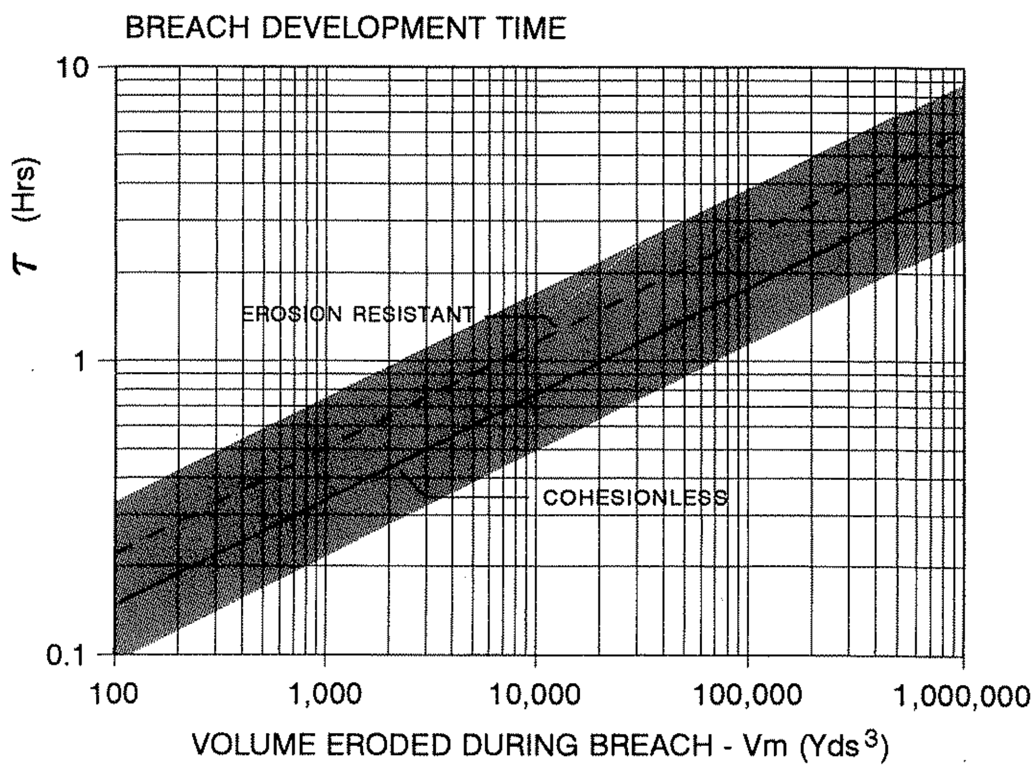

FIGURE 3 - ELAPSED TIME IN BREACH DEVELOPMENT

Gray tone areas have been added to Figure 3 to reflect the scatter in observed values[4] and the uncertainties involved in parameter estimation. In particular, some investigators have focused attention on the data compiled by MacDonald and Langridge-Monopolis as being a mixture of actual breach development times and times to drain the reservoir following onset of failure. This fuzziness in the data arises from the necessary use of eyewitness accounts for many of the observed failures. This has undoubtedly added to the variability of the data displayed in Figure 3 and biased the results. Thus, equations 5a and 5b may tend to overestimate the breach development time.

In addition, the MacDonald and Langridge-Monopolis[4] data are more representative of intermediate and large size dams. Extrapolation of their data to small dams appears to produce unrealistically short breach development times in some cases for small dams (6 feet to 15 feet high). A lower limit for the breach development time of perhaps 10 minutes for dams constructed of cohesionless materials and 15 minutes for dams constructed of erosion resistant materials seems reasonable.

Because of the uncertainties associated with the selection of the time for breach development, engineers should use a range of values to assess the sensitivity of the computed dam break flood peak discharge. The scope of sensitivity analyses are discussed in more detail in section 5.2.

An alternative procedure for estimating breach development time and the breach outflow hydrograph was developed by Fread[6]. In 1987, he completed the development of computer program BREACH for the numerical simulation of breach formation. This program can be used to compute the breach outflow hydrograph using the principles of hydraulics, sediment transport, soil mechanics, the material properties of the dam, and the reservoir storage and inflow characteristics. This program is generally considered to be the most analytically sound computational procedure currently available for estimating the breaching characteristics of earthen dams.

In summary, the three methods: Table 3 values; Equations 1 through 5b; and computer program BREACH; represent approaches to the estimation of breach parameters which are in common practice. For purposes of use with the Dam Safety Regulations, Chapter 173-175 WAC, only the latter two methods are recommended and incorporated into procedures for earthfill dams utilized later in this technical note.

2.2 ESTIMATION OF DAM BREACH PEAK DISCHARGE

Early work on estimating the flood peak discharge from a dam breach concentrated on establishing envelope curves for the largest observed dam break floods. Envelope curves (Figure 4) have been developed by MacDonald and Langridge-Monopolis[4], Costa[12], the Interagency Committee on Dam Safety (ICODS)[7] and others. While envelope curves are useful in identifying upper limits, they provide little information on what would be a reasonable estimate at a given project.

A significant improvement over envelope curves can be achieved by utilizing available computer programs such as DAMBRK developed by Fread[8] or HEC-1[9], a program developed by the Corps of Engineers. Both computer models utilize unsteady flow routing in combination with user selected breach parameters of width, sideslope and failure time to compute the breach outflow flood hydrograph. This methodology has been generally recognized as standard practice on large dams for over a decade.

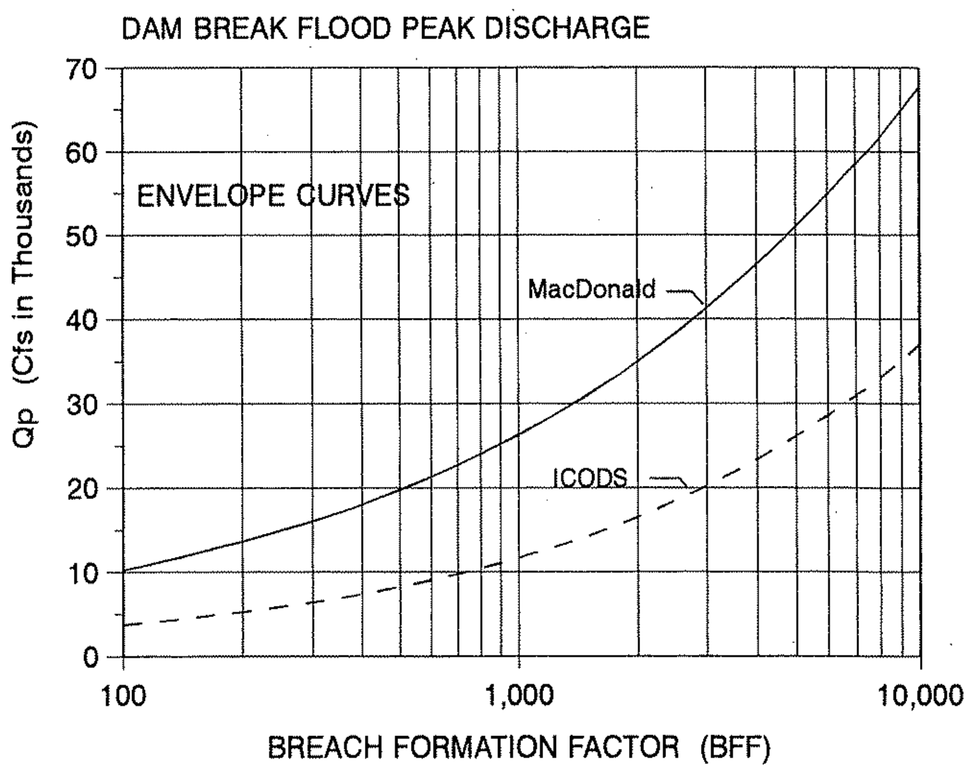

FIGURE 4 - ENVELOPE CURVES FOR DAM BREACH PEAK DISCHARGE

An alternative approach, suitable for many planning purposes, is given by Fread[3]. He developed an empirical equation based on numerous simulations with the DAMBRK model. Estimation of the peak discharge from a dam breach is computed as:

$$Q_p = 3.1\ WH^{1.5} \left[\frac{A}{A + \tau \sqrt{H}} \right]^3 \tag{6}$$

where:

Q_p = Dam breach peak discharge (cfs)

W = Average breach width (ft), $W = W_b + Z_b H$ (7)

H = Initial height of water (ft) over the base elevation of the breach

τ = Elapsed time for breach development (hrs)

A = $23.4\ S_a/W$ (8)

and:

S_a = Surface area of reservoir (acres) at reservoir level corresponding to depth H

The first component of equation 6 is seen as the standard weir equation with the width of the weir crest corresponding to the average breach width (W) and the head on the weir corresponding to the reservoir depth (H). This first component of equation 6 represents the peak discharge for an infinitely large reservoir where there would be no reduction of the reservoir level during erosion of the breach. The second component of equation 6 produces a reduction factor which accounts for the reduction in reservoir level during breach erosion. For a high dam with a relatively small reservoir, there can be a significant lowering of the reservoir level as water is released during formation of the erosional breach.

Utilization of the Fread equation above, in conjunction with equations 1 through 5b were used to generate tables to simplify estimation of dam breach peak discharges.

Tables 4a and 4b contain estimates of dam break peak discharges for overtopping induced failures of earthfill dams comprised of predominately cohesionless and erosion resistant materials, respectively. They represent hypothetical failures and were computed without consideration of natural flood inflow to the reservoir to initiate the failure. Spillway outflow may be added to the estimated dam break peak discharge, as deemed appropriate by the analyst, to approximate natural flood contributions.

These tables were computed based on embankment geometries where the slope of the upstream and downstream faces are 3H:1V and 2H:1V, respectively, and the crest width (C, in feet) is:

$$C = 2 + 2\sqrt{H} \tag{9}$$

As discussed previously, the most analytically sophisticated methodology currently available for estimating the breach outflow hydrograph is the computer program BREACH[6] developed by Fread. This program incorporates principles of sediment transport, soil mechanics and unsteady flow hydraulics to compute both breach dimensions and the outflow hydrograph.

TABLE 4A - DAM BREACH PEAK DISCHARGE ESTIMATES FOR DAMS CONSTRUCTED OF COHESIONLESS MATERIALS

DAM HEIGHT (FEET)	DAM BREACH PEAK DISCHARGE (CFS)									
	RESERVOIR SURFACE AREA (ACRES)									
	2	4	7	10	15	20	30	40	60	100
4	170	300	460	610	830	1010	1090	1110	1130	1140
6	280	470	730	960	1300	1620	2220	2760	2920	3020
8	380	640	990	1300	1770	2200	3010	3750	5100	5840
10	480	810	1240	1630	2220	2770	3780	4700	6400	9420
12	570	970	1490	1960	2660	3320	4520	5630	7660	11280
14	670	1130	1730	2280	3100	3860	5250	6530	8880	13070
18	850	1440	2200	2890	3930	4880	6640	8250	11210	16500
22	1020	1730	2650	3470	4700	5850	7950	9880	13420	19720
26	1190	2010	3070	4020	5450	6770	9200	11430	15510	22770
30	1350	2280	3470	4550	6170	7650	10390	12900	17490	25660
35	1540	2600	3950	5170	7000	8700	11800	14640	19830	29080
40	1720	2900	4400	5760	7800	9680	13120	16280	22040	32290
45	1890	3190	4840	6330	8560	10620	14380	17830	24130	35330
50	2060	3460	5250	6860	9270	11500	15580	19310	26110	38200

TABLE 4B - DAM BREACH PEAK DISCHARGE ESTIMATES FOR DAMS CONSTRUCTED OF EROSION RESISTANT MATERIALS

DAM HEIGHT (FEET)	DAM BREACH PEAK DISCHARGE (CFS)									
	RESERVOIR SURFACE AREA (ACRES)									
	2	4	7	10	15	20	30	40	60	100
4	120	200	300	410	560	700	950	1090	1110	1130
6	190	320	500	650	880	1100	1510	1880	2560	2950
8	260	440	670	880	1200	1500	2050	2560	3480	5150
10	320	550	850	1120	1520	1900	2590	3220	4390	6480
12	390	670	1020	1340	1830	2280	3110	3870	5270	7770
14	460	780	1190	1570	2130	2650	3620	4500	6130	9040
18	590	1000	1520	2000	2710	3380	4600	5730	7790	11470
22	710	1200	1840	2410	3280	4080	5550	6900	9370	13790
26	830	1400	2140	2810	3810	4740	6450	8010	10880	16000
30	940	1600	2430	3190	4330	5380	7310	9080	12330	18110
35	1080	1830	2780	3650	4950	6140	8340	10360	14050	20620
40	1210	2050	3110	4080	5530	6870	9320	11570	15690	23010
45	1340	2260	3430	4500	6090	7560	10260	12730	17250	25290
50	1460	2460	3740	4900	6630	8230	11160	13840	18740	27450

3. DOWNSTREAM ROUTING OF DAM BREAK FLOOD

Flood routing is the term used to describe the movement of a flood wave as it traverses a reach of channel. Of particular interest in flood routing are: the reduction of the peak discharge as it moves downstream (attenuation); the travel time of the flood peak between points of interest; the maximum water stage at points of interest; and the change in the flood hydrograph shape as it moves downstream.

These effects are governed by factors such as: the channel bedslope; the cross-sectional area and geometry of the main channel and overbank areas; the roughness of the main channel and overbank; the existence of storage of floodwaters in off-channel areas offset from active water conveyance areas; and the shape of the flood hydrograph as it enters the channel reach. These factors may be grouped as follows (Table 5) to indicate the relative amount of attenuation that may be expected.

TABLE 5 - FLOOD ROUTING ATTENUATION CHARACTERISTICS

SMALL ATTENUATION	LARGE ATTENUATION	CONSIDERATIONS
LARGE RESERVOIR VOLUME	SMALL RESERVOIR VOLUME	RELATIVE COMPARISON BETWEEN RESERVOIR STORAGE VOLUME AND STORAGE CAPACITY DOWNSTREAM CHANNEL AND FLOODPLAIN
SMALL CONFINING CHANNEL AND STEEP CHANNEL SLOPES	BROAD FLOODPLAIN AND/OR OFF-CHANNEL STORAGE AREAS AND MILD CHANNEL SLOPES	GENERALLY, SLOPES GREATER THAN ABOUT 1% ARE CONSIDERED STEEP
LITTLE FRICTIONAL RESISTANCE IN CHANNEL AND OVERBANK AREAS	LARGE FRICTIONAL RESISTANCE IN CHANNEL AND OVERBANK AREAS	PRESENCE OF SHRUBS, TREES, CROPS IN OVERBANK AREAS

3.1 <u>COMPUTATIONAL METHODS FOR ROUTING OF DAM BREAK FLOOD</u>

Computational schemes which can account for the physical characteristics of the channel reach and the hydrodynamics of flood wave movement are best suited for routing of dam break floods.

There are a variety of methods available for routing of the dam break flood through the downstream channel and floodplain. A simplified procedure suitable for many planning purposes has been developed by the USBR[10] based on observed dam failures. This procedure formed the basis for development of the generalized flood attenuation curves presented in Figure 5. These curves should be used conservatively, as they utilize generalized solutions to approximate the reduction in flood peak discharge with distance downstream of the dam .

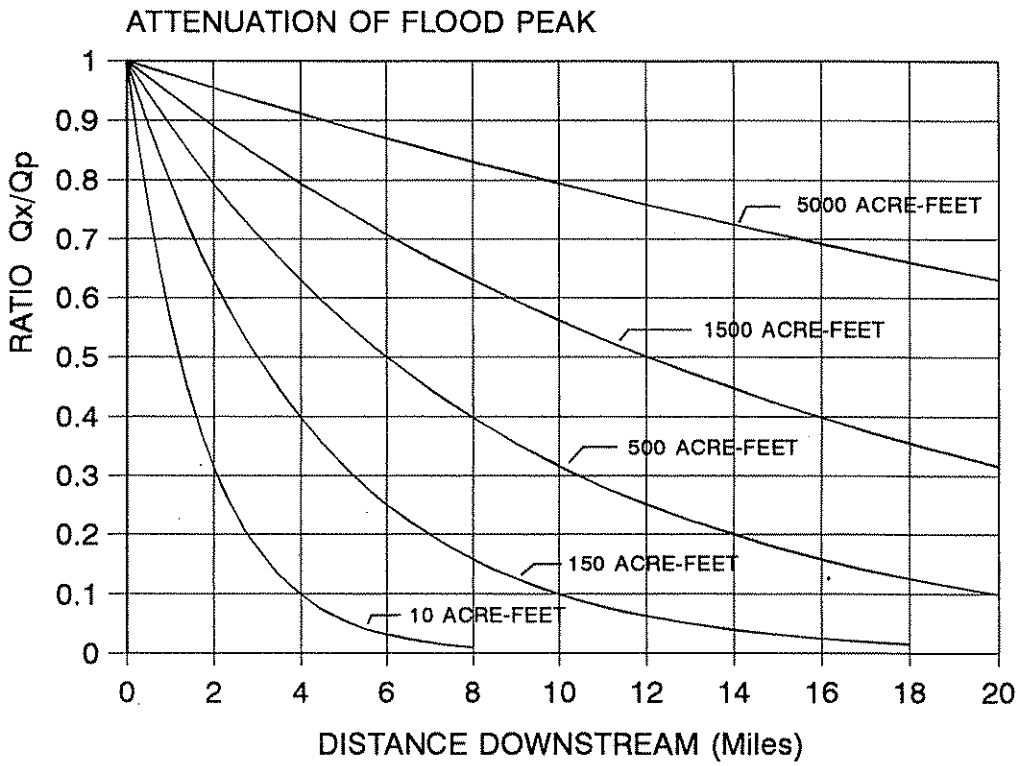

FIGURE 5 - GENERALIZED FLOOD ATTENUATION CURVES

The family of attenuation curves contained in Figure 5 are arranged according to reservoir storage volume (acre-feet). The attenuation is described in terms of the dam break peak discharge (Q_p) at the dam site and the peak discharge (Q_x) at some distance downstream.

More sophisticated routing methods, in increasing order of sophistication, include: hydrologic; diffusion; and hydraulic routing. Examples of these methods are listed in Table 6.

Flood routing should be continued to a point downstream where the dam break flood no longer poses a risk to life and there is limited potential for further property damage. Flood routing is usually terminated when the dam break flood enters a large body of water which could accommodate the floodwaters without a significant increase in water level or when the flood has attenuated to a level which is within the 100-year floodplain for the receiving stream. In the latter case, flood plain inundation maps may be available (through the Federal Emergency Management Agency (FEMA)) for use in inundation mapping in these areas.

When routing dam break floods in steep channels, care should be exercised to realistically account for the large magnitude energy losses produced by abrupt changes in channel geometry and alignment. Investigations by Jarrett[14,15,16] have shown that supercritical flow is uncommon in steep natural channels, particularly mountain streams. The irregularity of the channel geometry, presence of boulders and frequent changes in channel alignment cause large energy losses which generally restrict flow to the subcritical range. Artificially large Manning's n values are often needed to account for the increased roughness and energy losses posed by the above conditions.

TABLE 6 - FLOOD ROUTING METHODS IN COMMON USAGE FOR DAM BREAK ANALYSIS

FLOOD ROUTING METHODOLOGY	METHOD	COMPUTER MODEL OR REFERENCE SOURCE
HYDROLOGIC ROUTING	MODIFIED PULS	In HEC-1[9]
	ATT-KIN	SCS - TR-66[11]
DIFFUSION ROUTING	MUSKINGUM-CUNGE	in HEC-1[9]
	TWO DIMENSIONAL HROMADKA[13]	DIFFUSION HYDRODYNAMIC MODEL[13]
HYDRAULIC ROUTING	4 POINT IMPLICIT SOLUTION OF SAINT VENANT UNSTEADY FLOW EQUATIONS	DAMBRK[8]

4. INUNDATION MAPPING

The inundation map provides a description of the areal extent of flooding which would be produced by the dam break flood. It should also identify zones of high velocity flow and depict inundation for representative cross-sections of the channel. This information is standard output from many computer flood routing models and inundation maps may be developed utilizing cross-section and flood height data in conjunction with U.S. Geological Survey topographic maps.

For many planning purposes, a reasonable approximation of the inundation at a given location can be made using flood peak discharge information from Tables 4a or 4b, the attenuation curves in Figure 5, site specific channel cross-section data and representative flow velocities from Table 7.

TABLE 7 - REPRESENTATIVE VELOCITIES FOR USE IN ESTIMATING INUNDATION FROM DAM BREAK FLOODS

TYPE 1 MAIN CHANNEL - GRAVEL OVERBANKS - GRASS, PASTURE		TYPE 2 MAIN CHANNEL - GRAVEL, COBBLES OVERBANKS - IRREGULAR, BRUSH, SCATTERED SHRUBS		TYPE 3 MAIN CHANNEL GRAVEL COBBLES, BOULDERS OVERBANKS WOODED	
BEDSLOPE (ft/mi)	VELOCITY (ft/sec)	BEDSLOPE (ft/mi)	VELOCITY (ft/sec)	BEDSLOPE (ft/mi)	VELOCITY (ft/sec)
5	2.4	5	1.7	5	1.4
10	3.4	10	2.4	10	1.9
15	4.1	15	3.0	15	2.4
20	4.8	20	3.5	20	2.7
30	5.8	30	4.2	30	3.3
40	6.7	40	4.9	40	3.8
60	8.2	60	6.0	60	4.7
80	9.5	80	6.9	80	5.4
100	10.6	100	7.7	100	6.1
200	12.0	200	10.9	200	8.6
300	12.0	300	12.0	300	10.5
400 or greater	12.0	400 or greater	12.0	400 or greater	12.0

The cross-sectional area of flow required to pass the flood would be

$$A = Q_x/V \tag{10}$$

where:

A = Cross-sectional area of channel and overbank (ft^2) needed to pass the flood
Q_x = Flood peak discharge (cfs) at location x
V = Representative, average velocity (ft/sec)

Whether using the results of the simplified method above, or data from computer modeling, one should consider the potential effects of debris buildup and sediment transport. The inundation map should represent a conservative estimate of the consequences of a dam failure.

5. RECOMMENDED PROCEDURES FOR CONDUCTING DAM BREAK INUNDATION ANALYSIS

In recommending procedures for conducting dam break inundation analyses, it is reasonable that the sophistication and accuracy of analyses be commensurate with the scale and complexity of the dam and downstream area under investigation. For small dams situated above sparsely populated broad valleys, approximate methods are both adequate and economical. However, for large dams situated above populated areas on complex floodplains, sophisticated modeling and additional sensitivity studies are often needed to properly assess the consequences of a dam failure.

As a means of spanning this wide range of project characteristics, Table 8 has been prepared which identifies logical combinations of procedures which can be used to conduct the analysis. The table is arranged such that the simplified methods are indicated for small dams, and the more sophisticated methods are recommended for the larger dams. In all cases, the analyst should use conservative judgement and upgrade the analysis procedures when the situation warrants.

TABLE 8. RECOMMENDED PROCEDURES FOR CONDUCTING DAM BREAK INUNDATION ANALYSES

APPLICATION	BREACH DIMENSIONS	DAM BREACH PEAK DISCHARGE	DOWNSTREAM ROUTING	INUNDATION MAPPING
SMALL DAMS Height < 15 Feet	-----	TABLES 4a,4b	FIGURE 5	TABLE 7
	EQUATIONS 1 - 5b	EQUATION 6	FIGURE 5	TABLE 7
INTERMEDIATE SIZE DAMS	-----	TABLES 4a,4b	FIGURE 5	TABLE 7
	EQUATIONS 1 - 5b	EQUATION 6	FIGURE 5	TABLE 7
	EQUATIONS 1 - 5b	HEC-1	HEC-1	HEC-1
	EQUATIONS 1 - 5b	DAMBRK	DAMBRK	DAMBRK
LARGE DAMS Height $\geq$ 50 Feet	EQUATIONS 1 - 5b	DAMBRK	DAMBRK	DAMBRK
	BREACH	BREACH	DAMBRK	DAMBRK

5.1 RESERVOIR CONDITIONS AT TIME OF DAM FAILURE

While there are various applications for dam break inundation analysis, the common purpose is to assess the consequences posed by dam failure and release of the reservoir contents. Thus, the magnitude of reservoir storage is an important consideration in the analysis. Two reservoir conditions, normal pool and maximum storage elevation (dam overtopping), are usually examined in assessing the downstream consequences. Dam failure with the reservoir level at normal pool is often termed a "sunny day" failure and there may be little or no advance indication of the onset of failure. Conversely, a dam failure with the reservoir level at, or near, the dam crest is usually associated with an extreme flood event. In this case, several hours of advance warning may be available due to the obvious extreme meteorological conditions which produce the flooding. Therefore, these two reservoir conditions are important because they represent the potential for two different dam break flood magnitudes, and because the circumstances surrounding these two types of failure events may pose significantly different situations for consequences to life and property and warning of downstream inhabitants.

Another consideration associated with reservoir operation is the magnitude of the natural inflow and concurrent spillway releases at the assumed time of dam failure. These can also be important elements of the analysis and the values selected should be consistent with the hypothesized conditions at failure and the intended purpose of the analysis. The various applications and recommended procedures are discussed below.

5.1.1 Dam Failure at Normal Pool Condition

For a hypothesized failure at normal pool, it is reasonable to use quantities of reservoir inflow and outflow which are representative of the conditions or season(s) of the year at which normal pool occurs. If the HEC-1[9] or DAMBRK[8] computer model is used for the analysis, this is accomplished by including the natural inflow and outlet works/spillway outflow quantities as input parameters to the model.

As a practical matter, the resultant dam break flood for the normal pool condition is relatively insensitive to the magnitude of reservoir inflow and outflow because the inflow/outflow are typically very small by comparison to the dam break flood.
If Table 4a or 4b, or if equations 1 through 6 are used to estimate the dam break flood peak discharge, the natural outflow from outlet works or spillways may either be added to the dam break flood peak or discarded based on the judgement of the analyst as to the magnitude of outflow and the site specific considerations.

5.1.2 Dam Failure at Maximum Storage Elevation - Flood Condition

Dam failure during a flood produces a larger dam break flood than a failure at normal pool because of the larger quantity of stored water. Guidance for conducting dam break inundation analyses for flood conditions is more complicated than analyses for the normal pool condition because of the need to account for the magnitude of flood inflow and spillway outflow at the assumed time of failure. Issues related to dam failure analysis during flood conditions are discussed below.

Maximum Storage Elevation - WAC 173-175-030 defines maximum storage elevation to be "the maximum attainable water surface elevation of the reservoir pool that could occur during extreme operating conditions. This elevation normally corresponds to the crest elevation of the dam." Dam failure analysis for the flood condition is normally taken to be an analysis for failure due to dam overtopping. Thus, the reservoir level at the assumed time of failure would be at, or above, the dam crest elevation.

The exception is when the project can accommodate the Probable Maximum Flood (PMF) and freeboard would exist at the time the maximum reservoir level is attained. For this case, failure is assumed to occur at the time the maximum storage elevation is reached.

Reservoir Inflow/Outflow - There is some discretion allowed in the selection of an appropriate reservoir inflow and concurrent spillway outflow for dam failure analysis during flood conditions. As a strict academic interpretation, in order to initiate dam overtopping, the reservoir inflow should correspond to a flood larger than that used in the design of the project. The exception, would be if the project is capable of accommodating the PMF, then the PMF would be used as the inflow flood.

Adherence to this strict interpretation may, however, result in unnecessary time and expense in modeling the inflow flood and spillway releases - and not necessarily produce results which are superior to those produced by approximate methods. Alternative methods for accounting for the reservoir inflow/outflow are proposed in the following sections for use in the various applications. These methods have generally been found to produce acceptable results, particularly for small and intermediate size dams, while avoiding the time and expense of more sophisticated computer model analyses.

Applications of dam break inundation analysis for the case of failure during a flood are briefly discussed in the following sections. Guidance in selecting methods of analysis and appropriate procedures are also given.

5.1.2.1 Use in Downstream Hazard Classification Analysis

The determination of the appropriate downstream hazard class (see section 6) is not overly sensitive to the selection of the magnitude of the natural flood inflow at the time of failure. While the magnitude of inflow is a contributing factor, the release of the reservoir waters is usually a dominant consideration (all other considerations being equal) in determining the magnitude of the dam break flood and the downstream consequences. In addition, the downstream hazard classes represent a broad range of consequences and oftentimes, even crude methods of analysis are sufficient to indicate the appropriate classification.

Thus, the use of simplified dam break methodologies usually results in the same downstream hazard class as that determined by more sophisticated methods. Accordingly, approximate methods for incorporating the natural inflow/outflow into the dam break analysis are acceptable for use at small and intermediate size dams.

Experience in the Dam Safety Section indicates that the use of the 100 year flood peak discharge as the inflow quantity and concurrent spillway outflow generally yields results which are representative of a dam failure by overtopping and produces results that are within the range of accuracy of available methods of analyses. When using Tables 4a or 4b, or equations 1 through 6, the dam break flood can be computed by simply adding the natural spillway outflow quantity to the estimated dam break flood peak discharge.

The reader should be advised that situations will occur where more detailed accounting of reservoir inflow and outflow will be needed in conjunction with the HEC-1[9] or DAMBRK[8] computer models to determine the appropriate downstream hazard class. In these instances, sensitivity analyses are often warranted (see section 5.2) in addition to more sophisticated analyses.

The appropriate downstream hazard class is ultimately determined based on the more severe consequences of failure for the two reservoir conditions, normal pool and maximum storage elevation.

5.1.2.2 Use in Selecting Design/Performance Levels for Critical Project Elements

An important application of dam break inundation analyses is in the selection of design/performance levels for the design of critical project elements. In this usage, the dam break inundation analysis is used to assess the potential consequences of dam failure on life and property in downstream areas. The underlying philosophy is that the greater the hazard posed by a failure - the more stringent is the design criteria needed to provide an acceptable level of protection for public safety. Detailed procedures for utilizing dam break inundation analysis in the selection of design/performance levels is presented in *Technical Note 2 of the Dam Safety Guidelines*.

With regard to the selection of the magnitude of inflow to the reservoir and spillway outflow, the procedures outlined above for use in downstream hazard classification are generally acceptable for use in this application. In addition, the hierarchy of recommended procedures for conducting dam break inundation analysis displayed in Table 8 are compatible with procedures in Technical Note 2.

5.1.2.3 Use in Incremental Damage Analysis

Another important application of dam break inundation analyses is in conducting Incremental Damage Analyses. In these analyses, an assessment is made of the impacts of the dam break flood relative to the damage caused by the natural flooding which precedes it. This procedure can sometimes be used for determining the magnitude of an acceptable Inflow Design Flood (IDF) and sizing the emergency spillway. In general, it has application where a dam and reservoir are "small" relative to the watershed it occupies. In such "run of the river projects" the potential damages from a dam failure may be small relative to the magnitude of damages from natural flooding which can be produced in the tributary watershed. This methodology is discussed in detail in *Part IV of the Dam Safety Guidelines, Dam Design and Construction.*

For this application, the magnitude of inflow to the reservoir and spillway outflow are critical considerations. The reservoir inflow is usually based on rainfall-runoff modeling of the watershed and the spillway releases are based upon the proposed configuration and operation of the project's spillways. Because the objective of this type of analysis is to examine incremental increases in flooding and damages caused directly by the dam failure, only hydraulic routing methods, such as contained in the DAMBRK[8] computer model, are sufficiently sophisticated to be used in the analysis.

5.1.3 Dam Failure at Maximum Storage Elevation - Off-Channel Storage Reservoirs

Inflow to off-channel storage reservoirs is usually regulated by man-made controls, such as diversion channels, pumps, valves, etc. For purposes of conducting a hypothetical dam failure analysis at the maximum storage elevation, it is usually assumed that failure or misoperation of the inflow regulating mechanism(s) causes the reservoir level to reach the dam crest elevation. In this particular case, the magnitude of the dam break flood is relatively insensitive to the regulated reservoir inflow and outflow because the inflow/outflow quantities are usually very small compared to the dam break flood.

If either the HEC-1[9] or DAMBRK[8] computer model is used, the analysis is accomplished by including the natural inflow and spillway outflow quantities as input parameters to the model. If Table 4a or 4b, or if equations 1 through 6 are used to estimate the dam break flood, the natural outflow from spillways may either be added to the dam break flood peak discharge or discarded based on the judgement of the analyst as to the magnitude of outflow and the site specific considerations.

5.2 SENSITIVITY ANALYSES

In conducting a dam break inundation analysis, there are numerous sources of uncertainty. In hypothesizing a mode of failure, selecting breach dimensions and the time for breach development, assumptions must be made and parameters selected which directly affect the magnitude of the resultant dam break flood. In addition, dam break floods usually produce flooding at a scale unprecedented in the downstream valley. The great magnitude of the flood and the complexity in attempting to model the three dimensional flow results in uncertainties about the computed levels of inundation.

Fortunately, studies by Fread[3] have shown that "errors associated with the breach characteristics dampen as the flood propagates downstream. Also, the percent error in the computed flow depth is less than that for routed discharge, cross-sectional area and/or flow resistance". These error properties tend to mitigate the uncertainties involved in the many computational steps of the analysis. Nonetheless, where minor differences in the estimated flow depth and inundation area significantly alter the potential consequences to life or property, then sensitivity studies should be included in the analysis.

The sensitivity studies should address how alternative parameters for breach size, time of breach development, initial reservoir conditions, downstream channel and overbank roughness, etc., affect the computed flow depth in downstream areas.

In the final analysis, the parameters should be conservatively chosen after due consideration of the likely best estimates and how sensitive the final solution is to the parameters selected.

DOWNSTREAM HAZARD CLASSIFICATION

6. INTRODUCTION

Downstream hazard is defined as "the potential loss of life or property damage downstream of a dam from floodwaters released at the dam or waters released by partial or complete failure of the dam"[18] .

Downstream Hazard Classification does not correspond to the condition of the dam or appurtenant works, nor the anticipated performance or operation of the dam. Rather, it is descriptive of the setting in areas downstream of the dam and is an index of the relative magnitude of the potential consequences to human life and development should a particular dam fail.

The Downstream Hazard Classification is used for a variety of purposes in the Dam Safety Regulations Chapter 173-175 WAC, in the *Dam Safety Guidelines*, and in the internal operations of the Washington State Dam Safety Program. Uses include:

- A Reasonably Concise Indicator of the Relative Magnitude of the Downstream Consequences from Failure of a Given Dam
- An Index for Establishing General Design Requirements and Criteria
- An Index for Identifying those Dams where an Emergency Action Plan is Required
- A Management Tool for Allocating Time and Prioritizing the State Dam Safety Program Activities for: Construction Inspection; Periodic Inspection; and Compliance and Enforcement.
- A Classification System Compatible with National Criteria for Downstream Hazard Classification and Incorporation into National Databases on Dam Characteristics

7. DOWNSTREAM HAZARD CLASSIFICATION SYSTEM

The downstream hazard classification system adopted for use in Washington State is shown in Table 9. It is similar to systems in common usage in other State Dam Safety programs and has similarities to national hazard classification systems described in the *Recommended Guidelines for the Safety Inspection of Dams*[19] developed by the Army Corps of Engineers and the *Downstream Hazard Classification Guidelines*[17] developed by the Bureau of Reclamation.

In determining the downstream hazard classification of a given project, hypothetical dam failures should be evaluated for two reservoir conditions - normal pool level, and maximum storage elevation during flood conditions. The more severe consequences of failure for the two conditions should be used to establish the classification. In most cases, failure at the maximum storage elevation will produce the greater consequences. However, there are situations, such as where temporary use or recreational areas are located downstream of dams, where a sunny day failure at normal pool condition could pose the more severe consequences.

As outlined in Table 9, there are three principal considerations: the potential for loss of human life; the potential magnitude of property damage and corresponding economic losses; and the potential environmental damages. When comparing the relative consequences as listed in Columns 9A, 9B and 9C of Table 9, the most severe consequence will govern the selection of the hazard class.

As a final consideration, the potential for future downstream development should be investigated to determine if the classification might increase in the future. Each of these considerations is discussed below.

7.1 POPULATION AT RISK

The potential for loss of life is often the primary factor in determining the downstream hazard classification. For purposes of classification, the Population at Risk (PAR) is used to represent the potential for loss of life. This essentially corresponds to the number of people who would have to be evacuated from downstream areas in the event of a dam failure. Population at risk is defined in WAC 173-175-030 as - "the number of people who may be present in areas downstream of a dam and could be in danger in the event of a dam failure". This definition includes persons at permanent dwellings, worksites and at temporary use areas.

As general guidance, an inundation depth of 1 foot or more at a given dwelling, worksite or temporary use area can be used to indicate a hazard to life. Alternatively, the Bureau of Reclamation has published more detailed information on the hazards posed by various combinations of floodwater depth and velocity and has extensive commentary on classifying the downstream hazard in their publication *Downstream Hazard Classification Guidelines*[17].

With regard to estimating the population at risk below a given dam, it is common practice to use a value of 3 persons per inhabited dwelling[17]. Site specific information about the likely occupancy should be used at worksites such as water or wastewater treatment facilities, manufacturing or production facilities, farming operations, fish hatcheries, etc. and at temporary use facilities such as resorts, campgrounds and recreational areas. In all cases, conservative judgement should be exercised in estimating the areas that would be inundated and the population at risk.

TABLE 9 - DOWNSTREAM HAZARD CLASSIFICATION

DOWNSTREAM HAZARD POTENTIAL	DOWNSTREAM HAZARD CLASSIFICATION	COLUMN 9A POPULATION AT RISK	COLUMN 9B ECONOMIC LOSS GENERIC DESCRIPTIONS	COLUMN 9C ENVIRONMENTAL DAMAGES
LOW	3	0	Minimal. No inhabited structures. Limited agriculture development.	No deleterious materials in reservoir contents
SIGNIFICANT	2	1 to 6	Appreciable. 1 or 2 inhabited structures. Notable agriculture or work sites. Secondary highway and/or rail lines.	Limited water quality degradation from reservoir contents and only short term consequences.
HIGH	1C	7 to 30	Major. 3 to 10 inhabited structures. Low density suburban area with some industry and work sites. Primary highways and rail lines.	
HIGH	1B	31-300	Extreme. 11 to 100 inhabited structures. Medium density suburban or urban area with associated industry, property and transportation features.	Severe water quality degradation potential from reservoir contents and long term effects on aquatic and human life.
HIGH	1A	More than 300	Extreme. More than 100 inhabited structures. Highly developed, densely populated suburban or urban area with associated industry, property, transportation and community life line features.	

7.2 PROPERTY DAMAGE AND ECONOMIC LOSSES

Property damages would include damage to inhabited dwellings, commercial and production buildings, agricultural lands and crops, livestock, roads, highways and utilities and the associated economic losses both permanent and temporary. The intent, in considering the potential property damage and economic loss, is to identify the relative magnitude of losses against a broad scale of values. No attempt is made to assess actual fair market value or actual dollar losses.

Guidance is contained in Column 9B of Table 9 on how the relative amount of property damage and economic loss varies by hazard classification.

7.3 ENVIRONMENTAL DAMAGES

Consideration of environmental damages would address situations where the reservoir contains materials which may be deleterious to human or aquatic life or stream habitat. This applies to projects such as: domestic and agricultural waste lagoons; industrial waste lagoons; and mine tailings dams where the reservoir may contain trace amounts of heavy metals, chemical residues from ore processing, or large volumes of sediment in a loose or slurry condition.

Temporary damages to stream habitat are also to be considered. This would apply to streams with fisheries of regional significance where large scale channel scour and sediment deposition are likely to result from a dam break flood.

A review of Column 9C of Table 9 indicates the classification changes with the relative magnitude of the environmental damages. The most significant factors being the deleterious character of the reservoir contents and the duration of the effects - temporary or permanent.

7.4 CURRENT/FUTURE DEVELOPMENT

The downstream hazard classification should reflect the current downstream development and the associated consequences of dam failure.

However, it should be recognized that the future downstream development may increase the classification. This is important because the classification is used in *Part IV of the Dam Safety Guidelines* as an index for setting some of the engineering criteria for design and construction.

When using the classification in conjunction with *Part IV of the Dam Safety Guidelines*, it is advisable to investigate the effect that future downstream development may have in increasing the classification and increasing the minimum design standards/criteria at a given dam.

7.5 MULTIPLE DAMS

It sometimes occurs that two or more dams are constructed on a watercourse and the failure of the upstream dam may affect the downstream dam. If the failure of the upstream dam would not cause failure of the downstream dam, then the classification of the upstream dam is determined independently.

If the failure of the upstream dam would cause failure of the downstream dam, then the classification for the upstream dam must be as high or higher than the downstream dam(s).

7.6 MINE TAILINGS DAMS

The analysis of failure of mine tailings dams and the release of impounded slimes/tailings poses very difficult technical problems. Issues regarding the water content, soil grain size distribution, fluid properties and motility of the slimes/tailings further compound the already difficult technological problems associated with conducting the dam break inundation analysis. Features are available in the DAMBRK[8] computer model to approximate this phenomenon. However, the degree of success with this approach or any other method appears to be dependent upon the skill of the analyst and upon the similarity between the assumed properties of the slimes/tailings and the actual field conditions.

The Dam Safety Section will be open to methodologies and resultant Downstream Hazard Classifications which can be supported by reasonable analyses.

7.7 SOPHISTICATION OF APPROACH IN DETERMINING DOWNSTREAM HAZARD CLASSIFICATION

A review of Table 9 reveals that the five Downstream Hazard Classes (DHCs) span the entire range of potential consequences. Similarly, each downstream hazard class from DHC 3 to DHC 1A represents a range of consequences. Because of the broad nature of the classifications, the appropriate DHC can often be determined by windshield surveys and limited field work after the dam break flood and its attenuation have been determined.

In some cases, more extensive analysis of the dam break flood, inundation mapping and detailed field work will be needed to make a proper determination between two DHCs. Additional discussion on this issue is contained in section 5.1.2.1.

8. ENGINEERING REPORTS FOR DAM BREAK INUNDATION ANALYSES AND DOWNSTREAM HAZARD CLASSIFICATION

The computation/estimation of a dam break flood is dependent upon numerous characteristics of the dam, the mode of failure and the volume of storage at the time of failure.

Reports which discuss the findings from a Dam Break Inundation Analysis should address the following issues and list the pertinent parameters selected.

DAM BREAK FLOOD

- The Reservoir Level and Assumed Inflow at the Time of the Hypothetical Failure
- The Method of Estimating/Selecting the Breaching Dimensions and Characteristics for the Assumed Mode of Failure
- The Magnitude of the Estimated Dam Break Peak Discharge at the Dam Site and the Attenuation of the Flood Peak Discharge as it Propagates through the Downstream Valley

INUNDATION ANALYSIS

- The Travel Time of the Flood Wave to Various Locations in the Downstream Valley
- An Inundation Map Depicting the Areal Extent of Flooding
- Representative Channel/Valley Cross-Sections Depicting Flow Depth and Typical Flow Velocities

DOWNSTREAM HAZARD CLASSIFICATION

- A General Description of the Valley and Level of Development Downstream of the Dam
- The Method Used to Determine the Downstream Hazard Class

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